657

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Current discussion of papers sponsored by the Soil Mechanics and Foundations Division is presented as follows:

Number		Page
363	Field Penetration Tests for Selection of Sheepsfoot Rollers, by S.J. Johnson and W.G. Shockley. (December, 1953. Prior discussion: 518. Discussion closed)	
	Johnson, S.J. and Shockley, W.G. (Closure)	1
476	Studies of Bearpaw Shale at a Damsite in Saskatchewan, by Robert Peterson. (August, 1954. Prior discussion: 656. Discussion closed)	
	Mayer, Armand	3
477	Some Applications of Geology in Soil Mechanics and Foundation Engineering, by Parker D. Trask and H.B. Seed. (August, 1954. Prior discussion: none. Discussion closed)	
	Apfel, Earl T. and Goodman, Louis J.	5
	Gordon, Bernard B.	7
499	Performance of Soils Versus Chemical Properties, by C. Kinney Hancock. (September, 1954. Prior discussion: none. Discussion closed)	
	Huang, Eugene Y.	11
513	Foundation Conditions in the Cuyahoga River Valley, by Ralph B. Peck. (October, 1954. Prior discussion: none. Discussion closed)	
	Casagrande, Arthur	13
516	Engineering Properties of Expansive Clays, by W.G. Holtz and H.J. Gibbs. (October, 1954. Prior discussion: none. Discussion closed)	
	Dawson, Raymond F.	15
548	Foundation Treatment for Earth Dams on Rock, by Thomas F. Thompson. (November, 1954. Prior discussion: none. Discussion closed)	
	Fluhr, Thomas W.	19
550	Exploration Principles for Major Engineering Works, by W.R. Judd. (November, 1954. Prior discussion: none. Discussion closed)	
	Fluhr, Thomas W.	21

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Discussion of
"FIELD PENETRATION TESTS FOR SELECTION OF SHEEPSFOOT ROLLERS"

by S.J. Johnson and W.G. Shockley
(Proc. Sep. 363)

S.J. JOHNSON,[1] M. ASCE and W.G. SHOCKLEY,[2] A.M. ASCE.—The comments of Mr. Kantey in discussing the authors' paper are greatly appreciated. He inquires about the possibility of improving the agreement between Proctor needle penetration resistance and model foot penetration resistance (fig. 7) through the use of a size factor. It appears to the authors that the numerical values on fig. 7 reflect the influence of size of the penetration devices and any correction for size would change only the ordinate or abscissa values on the figure, thus shifting the curve as a whole but not bringing individual points into closer agreement. The values for Proctor needle penetration resistance shown on fig. 7 are data obtained on the compacted fill and not data on fill material compacted in cylinders. The reasons for scattering of data are believed those described in the paper under the heading, "Effect of Water Content." It is believed that less scatter in Proctor needle readings would be obtained in samples compacted in cylinders in the laboratory as suggested under "Conclusions." The scatter of data on fig. 7 is believed to be fairly moderate, especially at low loads, and it is considered that the problem of correlating the Proctor needle penetration resistance with the load on the model foot is a minor one.

Mr. Kantey also raises the question of computing the optimum sheepsfoot roller foot-pressure intensities by means of triaxial compression test results on the soil. Laboratory test data were available on the soil used in the Waterways Experiment Station test section in the form of families of curves of unconsolidated-undrained triaxial shear strengths over a range of moisture contents and densities.[3] These data were obtained on soil samples prepared in the laboratory at various compactive efforts. From these data shear strengths were determined for moisture contents and densities corresponding to two of the ranges shown on the load-penetration curves, fig. 9, as follows:

Moisture Content Per Cent	Unit Dry Weight Lb per Cu Ft	ϕ Degrees	Cohesion Tons per Sq Ft
18	103	15.6	0.90
21	100	7.7	0.75

1. Associate, Moran, Proctor, Mueser & Rutledge, New York, N.Y. (Formerly Chf., Embankment and Foundation Branch, Waterways Experiment Station, Vicksburg, Miss.
2. Chf., Embankment and Foundation Branch, Soils Div., Waterways Experiment Station, Vicksburg, Miss.
3. Effects of Size of Feet on Sheepsfoot Roller, T.M. 3-271, Report No. 6, Waterways Experiment Station, CE, June 1954.

The above-listed values were introduced into Terzaghi's bearing capacity formula for shallow circular footings[4] and the allowable bearing capacities were computed. For a moisture content of 18 per cent and unit weight of 103 lb per cu ft, the allowable bearing capacity was about 215 lb per sq in. Correspondingly, for a moisture content of 21 per cent and unit weight of 100 lb per cu ft, the bearing capacity was about 110 lb per sq in. If these values are compared with the load penetration curves on fig. 9 for areas having corresponding water contents, it may be seen that the computed bearing capacity value for a water content of 21 per cent is in good agreement with the model foot penetration resistance (areas 1 and 4), whereas for a moisture content of 18 per cent the computed value is somewhat lower than the load penetration curves (area 2). Plotting the computed bearing capacity values on fig. 8 shows good agreement with the curve of actually measured pressures on the sheepsfoot roller. Thus, it appears that bearing capacity equations may have sufficient value to represent a desirable starting point for selection of roller foot pressures in the field. It is recognized that values computed from these equations may not be precise, and field observation of roller behavior may be necessary in order to make final adjustments of roller foot pressures. The authors do not believe that these adjustments would be so large as to require a completely new roller on the job, as most sheepsfoot rollers can be adjusted to a wide range of foot pressures by use of different combinations of materials in filling the roller drums.

The use of bearing capacity curves for selection of optimum roller foot pressure is necessarily restricted to those cases where reliable shear strength data are available. In those cases where such data are not available, the authors suggest that the procedures outlined in their paper may be of considerable benefit.

4. Soil Mechanics in Engineering Practice, Terzaghi and Peck. John Wiley & Sons, N.Y., 1948, page 172.

Discussion of
"STUDIES OF BEARPAW SHALE AT
A DAMSITE IN SASKATCHEWAN"

by Robert Peterson
(Proc. Sep. 476)

ARMAND MAYER, M. ASCE.[1]—It has been especially gratifying for the writer to read Mr. R. Peterson's study on bearpaw shale; first because it reminded him of a very interesting trip to the damsite in May 1953 and, also, because it provided an opportunity to clarify some points which were not clearly fixed at the time of that visit.

As an over consolidated, more or less stiff clay, the bearpaw shale is very similar to the Paris "argile plastique," a layer of more or less plastic clay which underlays a large part of the Paris area, mostly east of the city. In the vicinity of Provins where it is recovered by means of typical subsurface mining operations it forms one of France's most important sources of fireclays. Very extensive laboratory studies of this plastic clay have been made and the results compare very closely with those given by Mr. Peterson for the soft and medium shale.

		bearpaw shale		Argile plastique
		soft	medium	
water content		29-36	25-31	25-28
Atterberg limits	L.L	95-135		105-123
	P.L.	20-25		26-33
	P.I.	60-120		75-95
Permeability (cent.sec.)		10^{-9}-10^{-11}		10^{-9}-10^{-10}
Swelling pressure (ton/sq ft)		.3-1	1-2	1-4
Shear strength (quick)	angle of internal friction	12°	15°	12°
	cohesion	15-35	35-55	10-40

In France, as in the Saskatchewan area reported by the author, the deposit has been compressed under an ancient loading very much heavier than that which exists at present. As a consequence of this and also the relative impermeability of the material the residual effective stresses are much greater in magnitude than those which could be produced by the existing overburden. This is particularly noticeable in the mining operations where the excavation provides an opportunity for the release of the excess pressure through expansion. In many places the tunnels are completely closed despite the substantial timbering used. That this is an expansive movement rather than a flow has been clearly demonstrated on numerous occasions where new tunnels have penetrated older, blocked tunnels. At these intersections the older timbering has been found completely encased in the clay but without voids or other indications of displacement.

1. President of the Cement Industry Research and Development Center, Paris, France.

Many tests have been made in this clay, that could probably be repeated in the bearpaw shale. Even an attempt was made before the war to measure in situ the swelling pressure of the clay. A cylindrical hole was dug into the clay and a rubber hose placed so as completely to fill the empty space. The hose was filled with water, connected to a manometer and left there for several months. During that time the tunnel section, where the test was made, was firmly braced, so as to prevent the plastic flow of the clay and let the pressure act on the hose. A leakage of the connection between the hose and the manometer stopped the test as the pressure had already exceeded 8 ton/sqft, that is, distinctly exceeded the weight of the overburden. It was not possible to repeat the test, on account of the war, but a proof had been brought of the fact that the pore pressure in that natural clay, by far exceeded the pressure of the overburden.

This was also apparent in the laboratory consolidation tests but had not to the writer's knowledge been checked in the field.

Similar results were obtained in another part of the clay deposit, south of Paris, during the preliminary tests for the construction of a subway station. The tests had shown that the swelling pressure, in this particular area would be 4 tons/sqft. The contractor at that time decided he knew better and that such a pressure was impossible so near the surface. Experience showed the slabs on which the track was layed blew up, then the sides broke in, and the stretch had to be completely reconstructed.

Mr. Peterson also mentions the existence of numerous slickensides and joints. The same can be observed in the mines near Provins. But it is the writers belief that, as in Saskatchewan, these slickensides are a result of the excavation and that they do not preexist in the virgin clay.

The main difference between the layers in Saskatchewan and in Provins is that the Saskatchewan river has cut into the clay layers, thus causing a progressive decompression towards the unconfined areas which results in a increment of the plasticity of the clay and numerous landslides. The plastic clay very seldom shows at the surface in the Paris area. Where it does the same kind of slides also happen due to the same cause.

But the experience gained in this country shows that once the material is well known and its properties made clear, the plastic clay does not present any particular danger. It has been used as a core for an earthdam without any harm, although great difficulties were encountered for placing it. It has been dug thru by the Paris subway several times. Tests are necessary to check the nature of the clay at the particular spot.

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Discussion of
"SOME APPLICATIONS OF GEOLOGY IN SOIL MECHANICS AND FOUNDATION ENGINEERING"

by Parker D. Trask and H.B. Seed
(Proc. Sep. 477)

EARL T. APFEL[1] and LOUIS J. GOODMAN,[2] A.M. ASCE.—The paper by Trask and Seed explores a very fruitful topic for both geologists and soils engineers. The collaboration of engineering geologists and civil engineers with soil mechanics specialization has made it possible to shorten the time of subsurface explorations, adapt the most appropriate method to the problem of investigation, to design a program of laboratory testing which will have maximum significance, and the soils engineer to cooperate with the design engineer in an effective measure.

The methods of the geologist in the writers' case parallel those reported in the paper under discussion, but there are some significant differences between the areas used for illustration by Trask and Seed and the areas in which the present writers have worked. Also the writers have used geophysical methods to supplement geological observations and drill data, in a number of cases with economies both in time and cost.

The areal differences are those produced largely by glaciation, periglacial conditions and by a succession of glacial lakes and temporary drainage ways. The deposits associated with the action of glacial ice, fluvial-glacial and glacial-lacustrine waters constitute most of the overburden in the writers' area as well as in most of the mid-west and northeast. The fact that glaciation was recurrent with the repetition of depositional conditions presents certain hazards to the acceptance of "refusal" on borings as an index of substantial bearing capacity.

Peat deposits are wide spread, some of these deposits are buried under what might normally be considered a full depositional sequence of granular materials. Plastic lacustrine clays are found similarly buried and poorly compacted silt underlies many square miles of former glacial lakes.

The direct deposit from glacial ice—glacial till—varies with the part of the ice from which it was deposited, whether from the base of the ice or near the surface, and whether the till was subjected to overriding by subsequent glacial ice during later glacial stages. Consolidation tests have shown the till to range from the compaction produced by present loading to that which would have been produced by the surcharge of an ice mass about 4000 feet thick standing over the material.

1. Chairman, Dept. of Geology, Syracuse Univ.; Director, Geo-Research Div., Syracuse Univ. Research Inst.; Geology Consultant, Subsurface Surveys, Syracuse, N.Y.
2. Associate Prof. of Civ. Eng., Syracuse Univ.; Soils Engr., Geo-Research Div., Syracuse Univ. Research Inst.; Soils Consultant, Subsurface Surveys, Syracuse, N.Y.

Glacial deposits commonly contain boulders in the till, or granular materials may contain cobble and boulder inclusions either in single pieces or in beds. Test drilling might show solid rock at depths much above the bed rock, but such rock would be only large fragments. Boulders up to 15 feet and more in diameter are common in some areas in which the writers have recently worked; special care must be exercised in recognizing boulders as contrasted with bedrock under the overburden.

The particular keys to the solution of the geological problems mentioned above are a knowledge of geologic history of the glacial period, a sense of the importance of topography in controlling drainage, and the availability of methods of distinguishing between glacial drifts of different ages. Engineering geology in the area formerly covered by continental glaciers is to a large extent glacial geology.

The refraction seismograph has been a tool of pronounced value in subsurface exploration in glaciated areas; it has also been used very effectively in areas where residual cover rather than glacial or alluvial cover lies over the bed rock. A correlation of seismic velocities with type of subsurface material is used to marked advantage in extending information from a limited number of borings over a wider area. It has not yet been possible to interpret with assurance the bearing capacities of subsurface materials from seismic data, but research directed toward identifying such relationships promises a degree of success.

The geological use of the refraction seismograph calls for knowledge of the geological characteristics both of the different types of overburden and of the underlying bed rock. The use of pilot borings or check-drilling is deemed essential for the proper interpretation of refraction seismic data. The soils engineer and geologist cooperatively studying the stratigraphic section revealed by the boring, and the engineering attributes of the materials as revealed by classification and other laboratory tests, are in position to better understand the significance of seismic records from the area and to recognize significant changes which may indicate the need for additional check borings.

Another attribute of the application of Geology to foundation engineering would be in the identification of buried river valleys which may occur, especially in glaciated areas. Regional studies are indicative of the presence of such valleys but further precise determination of depth, width, slope of valley sides as well as the valley location are dependent upon local exploration. The refraction seismograph, used with an appropriate number of check-borings is an effective geologic aid for these determinations.

A knowledge of the character of the bed rock is particularly important where foliated metamorphic rocks or steeply dipping sedimentary rocks are involved. Seismic velocity may vary greatly between directions along the strike and normal to the strike of the foliation or bedding. Because computations of depth of the overburden are based upon relative seismic velocities of overburden and bed rock, the range of seismic velocities in bed rock is necessarily determined. But there are other geologic features of concern to the foundation engineer which the refraction seismograph identifies or may indicate as subject to further checking. Some of these are faults, excessively fractured zones, collapsed rubble, and cavernous rock. The geological recognition of the possibility or probability of the occurrence of any of these conditions is a basis for the design of an exploration program which will determine their importance in the particular area being studied. The foundation engineer can then adjust his design so as to use safely the bearing materials on which his structure rests.

Close collaboration of the geologist, soils engineer and foundation design engineer have marked advantages in effecting economies in overall foundation costs as well as in avoiding a number of hazards to foundation stability. The fact that the writers have recently been consulted in several cases where structures have settled with the development of hazards to further use, is evidence that better subsurface information can pay dividends when it is properly used by the design engineer. His utilization of the resources of the geologist and soils engineer may well relieve him of much of the concern over the effect of subsurface conditions on his design. This discussion is submitted in the hope that such cooperative studies may expand in number and importance.

BERNARD B. GORDON,[3] A.M. ASCE.—Introduction.—The authors have presented an excellent general treatment of the relationship between engineering geology and soil mechanics and have demonstrated very clearly how cooperative effort between these two related fields can best serve the engineering profession. The writer would like to illustrate just how such a collaboration resulted in major economies in a large engineering project just recently completed in the Bay Area.

East Bay Sewage Disposal Project

This $23,500,000 project which was designed and built under the direction of the East Bay Municipal Utility District has been described by Mr. R.C. Kennedy, M. ASCE, Chief Engineer of the District.[4] The project involved approximately 22 miles of intercepting sewer, a 15,000 foot outfall sewer, and a large treatment plant. This area is included in the authors' figures 1, 3, and 4. The information from which these figures were derived stemmed in part from the results of borings and soils laboratory tests performed by the Utility District for the Sewage Project.

Early Studies

Studies for the Sewage Project made prior to 1946 indicated that the Utility District would face difficult foundation problems for long stretches of the intercepting sewers, along all of the outfall sewer, and for most of the structures in the treatment plant. It was well established that over most of the route bedrock would be out of reach.

Route Selection

The Utility District was required to follow routes that would best serve existing sewer outfalls and would permit of easiest disposal of treatment plant effluent. Elevations of sewers and structures were set by hydraulic and other conditions and it was seldom possible to select either the route or the profile elevation from foundation considerations. It became the Utility District's problem therefore, to determine the general soil profile at the various locations and to select a foundation type that would provide the best support to the least overall cost.

3. Senior Civ. Engr., East Bay Municipal Utility Dist., Oakland, Calif.
4. "Sewage Disposal Project Cleans Up San Francisco Bay," Civil Engineering, June 1953, pp 46-49.

Soil Studies

The initial steps in the program consisted of studying old maps of the area and records of exploration and construction made available by other organizations. Then a series of soil borings were drilled to various depths at rather long intervals along the route of the sewers and at the location of the major structures, and from the results there was determined a general soil profile similar to that shown by the authors. A laboratory study of soil strengths revealed the presence of a preconsolidated crust at the base of the Bay mud. This crustal layer had been preconsolidated to pressures of from 2 to 4 tons per square foot and apparently would have ample strength to support the construction. The choice of foundation support would depend, of course, upon the relationship between the elevation of the base of the structures and the surface of the crust. The District had three choices: (1) supporting directly upon the firm crust; (2) removal of the Bay mud to the surface of the crust and replacement with a suitable material such as compacted clean sand; or (3) driving piles into, but not through, the crust.

Pile Loading Tests

Supporting capacities of both timber and cast-in-place concrete piles were determined by pile loading tests in the field. The District learned that for this preconsolidated material:

(1) pile formulae of the Engineering-News-Record type were ultraconservative and in many respects not even reliable.

(2) for the District's purposes the Merritt sand, the Posey clay and the top of the San Antonio formations would all be within practical reach for piling.

(3) for timber piles with an average diameter of 12 inches, penetration of 13 feet into the crustal layer resulted in a safe bearing capacity of 20 tons.

(4) for cast-in-place concrete piles with an average diameter of 13 inches an average penetration of 15 feet into the crust resulted in a safe bearing capacity of 40 tons.

Profiles of Soundings and Borings

Because of the widespread area in which the working cost of exploration became an important factor, it was necessary to devise an economical balance to get maximum return. By this time the areal pattern of the soil profile had been determined. It was now desirable to outline the actual profile of the top of the preconsolidated crust. As the authors have pointed out, mud was as much as 100 feet deep in the ancient valleys while in the flat areas between the valleys it was only 10 to 20 feet deep. In some locations the mud was completely absent and no foundation problems were anticipated. The District finally evolved a pattern of investigation wherein it relied upon soil borings spaced at about 1,000 foot intervals supplemented by soundings at intermediate spacings. The borings cost from $3 to $4 per lineal foot as compared to 25¢ to 50¢ per lineal foot for soundings, so the District was able to use the latter as close as 100 foot on centers. Where the mud was particularly deep or the structure of unusual importance borings were made at closer intervals. Data determined thusly were used to plot soil profiles.

Outfall Sewer

To illustrate the District's procedure on a key unit of the system the writer refers to the outfall sewer which runs from the treatment plant to a point some 15,000 feet to the west. This line crosses the area shown on the author's figures no. 4 and no. 5. The first 9,000 feet was along the filled land where the 108" inside diameter reinforced concrete pipe was practically at the surface and had to be supported the full length on timber piles.

From there the sewage drops down a vertical transition structure into 6,000 feet of 96" inside diameter pipe which was laid under water directly on the preconsolidated crust at the bottom of a channel dredged to the base of the mud.

In the contract drawings and specifications the District presented for the contractor's information supplementary drawings containing locations and logs of borings as well as profiles of the mud and preconsolidated crust as determined by the soundings.

There was also shown the estimated tip elevation of the piles based on the known elevation of the top of the firm crust and the required penetration into the crust. This information was provided for bidding purposes only and was to be verified in the field by driving test piles in designated locations. To keep the bid items flexible the District did two things: (1) set up as bid items separate lineal foot prices for furnishing timber piles in two categories of length and, (2) set up a series of bid items for installing piles of specific lengths at a price per pile for that specific length. Estimated pile lengths were based on careful and scientific studies and as a result the final driving records checked quite well with the predicted values.

Economics

The savings that resulted from the practices just described were truly remarkable. By using the knowledge obtained of the existence and profile of this preconsolidated crustal layer it was possible to reduce the total length of pile-supported sewer from the 9.6 miles first estimated to only 4.5 miles. Piled length of the outfall sewer makes up 1.8 miles of this. Secondly, the accurate determination of the amount of penetration required to develop the required safe bearing capacity of the piles made for very large savings on pile driving costs. During the course of the construction the District made a number of determinations of the length of timber piling which would have been required by the standard Engineering-News-Record formula and found that a minimum penetration of 25 feet would have been required except in those few areas where the Merritt sand was encountered. In the latter areas it was practical and economical to drive to refusal. Using the Engineering-News-Record formula would have increased the average length by 12 feet and for the 4.5 miles would have required an extra 100,000 lineal feet of piling. At an approximate cost of $2/lineal foot per timber pile installed there was a saving for this one item alone of over $200,000. The total cost for the foundation investigation, including not only borings and consulting services by contract but also District labor and equipment, amounted to approximately $150,000 or about 0.7% of the total project construction cost. In other words, the savings which resulted from the reduced length of piling in the actual piled sections of the sewers alone amounted to more than the total cost of the investigation. The major savings were obtained, however, by eliminating complicated and expensive foundations over much of the total length of sewers and for many of the major structures included in the project. The aggregate amount of these savings has not been accumulated as yet, but the indications are that it will be many times the $200,000 just described. In all on this project the District spent the following on piling in round numbers:

Interceptors	$290,000
Outfall	208,000
Structures	186,000
TOTAL	$684,000

(Which was 3.4% of total construction costs)

Summary

To summarize, savings were of four categories:

1) reduced length of piles over just the pile-supported length of sewers.
2) complete elimination of piles over other sections of sewers.
3) reduction of future maintenance by elimination of or major reduction in differential settlement of structures.
4) in addition a very large but intangible saving resulted from bid prices which ran consistently low as a result of the information made available to the contractor.

This investigation is proving of increasing value to the East Bay area in general because the results have been made available to other agencies and have thus contributed to a general increase in knowledge of the local soils and geology. The sewage plant and interceptors have been in operation for over a year now and constant checking by levels has failed to reveal any noticeable settlements.

Discussion of
"PERFORMANCE OF SOILS VERSUS
CHEMICAL PROPERTIES"

by C. Kinney Hancock
(Proc. Sep. 499)

EUGENE Y. HUANG,[1] A.M. ASCE.—Based on data obtained from laboratory tests and field observations, Professor Hancock has found a relationship between the silica-sesquioxide ratio of a soil and the performance of the soil when it was used as a roadbed material. Consequently the soil colloid silica-sesquioxide ratio has been proposed as a supplementary index to roadbed performance.

Inasmuch as the silica-sesquioxide ratio is determined for colloidal clay which is known to be the most chemically active fraction of a soil, there is no doubt but what this ratio is to a certain extent an index of some practical importance. In addition to the references enumerated by Professor Hancock, this relationship is revealed by the findings of Professor Winterkorn which were disclosed in 1935 and 1936.[2] In his studies on the surface-chemical factors influencing the engineering properties of soils, Professor Winterkorn has shown that the silica-sesquioxide ratio of several soils has a definite relationship with their physical constants, tensile and compressive strengths, and other physical characteristics and properties. It appears, however, that the presently available data regarding this relationship is far from sufficient to develop a general conclusion applicable to the engineering use of soils. Much more research along this line certainly would be necessary to the establishment of a useful correlation. In this connection, the writer believes that the colloidal behavior of a soil is probably dependent largely upon a few basic factors such as clay minerals, exchangeable bases, organic matter, and the like, to which the silica-sesquioxide ratio is more or less related, and that the silica-sesquioxide ratio or the double percentage ratio ($2\%SiO_2/\%R_2O_3$), as proposed by Professor Hancock, is probably not in itself alone a complete criterion and therefore can only be considered in relation to a number of other factors for dealing with roadbed soil problems.

In regard to the performance of a roadbed, which is dependent upon a combination of circumstances, the writer cannot accept the idea that either the silica-sesquioxide ratio as shown in Figs 1 to 3 or the physical constants (including Atterberg limits and shrinkage factors) of the roadbed soil alone may be used for its evaluation. The basic requirements for satisfactory roadbed soil materials are (1) sufficient mechanical stability to resist

1. Research Assistant Prof. of Civ. Eng., Univ. of Illinois, Urbana, Ill.
2. "Adsorption Phenomena in Relation to Soil Stabilization," by Hans F. Winterkorn, Proceedings of Highway Research Board, Vol. 15, 1935, pp. 343-355. "Surface-Chemical Factors Influencing The Engineering Properties of Soils," by Hans F. Winterkorn, Proceedings Highway Research Board, Vol. 16, 1936, pp. 293-308.

displacement under load under expected weather conditions and (2) ability to withstand a certain amount of volume change due to fluctuations in moisture content and temperature. An unsatisfactory roadbed may result from undesirable soil material, inadequate compaction, a high water table, an insufficient drainage system, frost heaving and frost boils, or a combination of such things. Failures may also occur, however, from an inadequate road structure, such as insufficient thickness or poor quality of surface course. In most cases, a combination of several factors with differing degrees of importance is what causes a roadbed to develop defects. The precise cause of a "good" or a "poor" roadbed in a given instance can, therefore, be determined only after a thorough observation of site conditions and a careful examination of construction records, plus, if necessary, a detailed laboratory study of the soil material. It seems very unlikely that roadbed performance can be directly correlated with the silica-sesquioxide ratio, a value which is associated only with the colloidal fraction of the material in the roadbed and which is in no way a measure of the many other important variables known to affect performance.

In typical highway construction the roadbed material is composed of a mixture of soil and aggregates, the texture of which ranges from colloidal clay to coarse gravel. From the standpoint of stability, as far as the roadbed is concerned, primary consideration should be given to the source of resistance, both cohesion and internal friction, in the soil mixture. The granular fraction is assumed to provide an internal skeleton of grains of sand and larger particles touching each other or interlocking with each other to furnish internal friction while the soil fines act as a filler and cementing medium to bind all the fractions into a mass. The load-carrying ability of a soil mixture depends upon the strength and nature of its individual particles and probably even more on the mechanical properties of the system or arrangement of the particles. Since the Atterberg limits and shrinkage factors indicate a soil's capacity to develop cohesion and the degree of volume change with varying moisture content, they are often used as qualitative tests for the soil fraction passing the No. 40 sieve in selecting the types of soil which are suitable to be used as roadbed material as well as for determining the amount of binder soil which is necessary to supply adequate cohesion without overfilling the voids between the granular particles of the mixture. The structural strength and stability of a roadbed, under most conditions, are dependent upon both its cohesion and internal friction. A criterion for roadbed performance evaluation or prediction can hardly be considered entirely reliable if it is based solely upon the physical constants of the finer fractions. It may be interesting to note from Table I of Professor Hancock's paper that the amount of material retained on the No. 40 sieve ranged from 26 to 57 per cent, indicating the granular character of the mixtures, the effect of which, according to the writer's opinion, should not be overlooked in evaluating roadbed materials.

Discussion of
"FOUNDATION CONDITIONS IN THE
CUYAHOGA RIVER VALLEY"

by Ralph B. Peck
(Proc. Sep. 513)

ARTHUR CASAGRANDE,[1] M. ASCE.—In his brief paper, Professor Peck succeeded in condensing a wealth of valuable information on the foundation conditions in the Cuyahoga Valley where the design of foundations for heavy loads poses many problems.

Of particular significance is the observation that the bearing capacity of friction piles in the overconsolidated clays is much smaller than what would be expected from the shear strength of the clay in its undisturbed state; that the developed friction along the embedded portion of the pile is more nearly equal to that of the completely remolded clay. The writer has made similar observations in overconsolidated clays at other locations, and he is inclined to generalize the hypothesis suggested by Professor Peck that such clays fail to reconsolidate after pile driving has remolded the adjacent clay. In the absence of any proof to the contrary, based in each case on thorough soil investigations and pile load tests, the designer should assume that the effective friction along the embedded portion of a pile is only equal to the strength of the clay when consolidated under the now existing overburden load and which is, for an average clay, equal to about one-fifth of the effective overburden stress. However, even this rule would give skin friction values which are too high for the clays underlying the Cuyahoga valley. It would seem in this case that the high excess hydrostatic pressures in the clay which were observed by Professor Peck, are an additional important factor in preventing reconsolidation even to the low strength of a normally consolidated clay.

Professor Peck points out that it has been general practice in this area to use pile foundations, with the piles driven through the relatively compressible silty and sandy alluvial deposits overlying the clay. If the bearing capacity of the piles driven through this deposit into the clay is properly evaluated, settlements are for practical purposes eliminated because the clay itself has already been preloaded during its history to stresses far greater than the stresses caused by the structures.

Since the purpose of piles in this area is chiefly to eliminate the settlements caused by the compression of the alluvial materials overlying the clay, preloading a building site appears to be a particularly simple and effective operation which would result in much less costly spread foundations in lieu of pile foundations. Therefore, when Mr. E.W. Hollister, of the Austin Company, consulted the writer regarding the proposed pile foundations for a heavy silo which The Austin Company was designing for the National Sugar Refining Company, the writer proposed to preload the site and to eliminate piles entirely. The foundation mat for this silo was designed as a 65 ft

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square, 5 ft thick reinforced concrete slab with its surface at ground level. After completion of this slab it was loaded with about 10,000 tons of gravel representing a load 50 per cent in excess of all dead and live load of this structure. The settlements under this temporary surcharge developed almost as quickly as the load was applied and then stopped, as was to be expected for the type of subsoils. The foundation mat settled a total of between 2 and 3 inches, as compared to settlements of between 2 and 5 inches which the writer had anticipated.

The 120 ft high, 40 ft diameter silo and appurtenant structures were completed in 1953. Filling of the silo began late in 1953. During the summer 1954 it showed, under a full load of 3300 tons of sugar, 5/16 inch settlement and upon subsequent removal of all load, the silo rebounded about 1/32 inch. Probably this settlement is due to time effects in the thin organic layers contained in the alluvial sand stratum, which at this site ranges between 40 and 60 ft in thickness. But possibly it reflects some consolidation of the underlying clay. The order of magnitude of these additional settlements is probably the same as that of a pile foundation, with friction piles in the clay, taking into consideration the elastic compression of the piles. Perhaps Professor Peck could add, in his Closure, information on the settlements which heavy structures on friction piles in the clay have experienced in this area.

Discussion of
"ENGINEERING PROPERTIES OF EXPANSIVE CLAYS"

by W.G. Holtz and H.J. Gibbs
(Proc. Sep. 516)

RAYMOND F. DAWSON,[1] M. ASCE. —Mr. Holtz and Mr. Gibbs are to be congratulated on a very fine paper on an important subject that has been neglected far too long. The expansive properties of clays and their effect on structures are closely tied in with climatic conditions, thus making it difficult to set any limit on the properties of soils in determining their swelling characteristics. Under conditions of uniformly distributed rainfall many clays may behave satisfactorily, but under conditions of long periods of dry weather the same clay will exhibit enormous expansive properties.

The authors have given a number of laboratory tests in identifying the expansive clays and have carefully pointed out the limitations of each of the tests. This information will be extremely valuable to the engineering profession; for in the past many people have suggested various tests as the solution in determining the expansive properties of clays, whereas in actuality certain other clays which have enormous swelling properties will not reveal this characteristic by a single test. The authors also point out the difficulties of having certain tests made and the fact that these cannot be used as routine tests in the engineering profession.

The free-swell test is very interesting and helpful. This should enable engineers to add another simple test to determine the possibilities of expansion in clay soils. In making the free-swell test it has occurred to the writer that perhaps it would be desirable to specify a time limit before taking the final reading. It has been our observation that after a few hours there has been considerable more swell than at the time the material was poured into the water. Furthermore, although Holtz and Gibbs suggest that free-swell values of below 100 %may not cause trouble and those having a free-swell value below 50 per cent very seldom exhibit appreciable volume changes, a number of Texas clays with free-swell values in the range of 50 per cent have caused considerable trouble due to expansion. It is believed that this expansion is due to the combination of climatic conditions and the soil rather than to the actual free-swell characteristics of the soil. Therefore, while limits that are fairly high may be set for one area, in other sections having more severe climatic conditions, lower values must be used. In the opinion of the writer, the free-swell test combined with the Atterberg test will provide the most satisfactory indicators that the engineer will have available to use on many jobs. It is also believed that any attempt to set limits for these tests must be coupled with the climatic condition which exists in the area.

The load expansion tests are interesting and warrant further study, particularly in relation to Skimpton's "Activity" value for the soils. It is a well-established fact that, as expanding clays are permitted to swell, the resulting pressures are greatly reduced. Figure No. 15. shows the swelling pressures that were developed at different moisture contents and at different

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percentages of expansion in a series of tests conducted at The University of Texas. The soil was a silty clay from the Taylor marls of Central Texas and had a low "Activity." Its physical characteristics are:

Liquid Limit: 50	Specific Gravity: 2.74
Plasticity Index: 31	Activity: 1.09

The optimum moisture at maximum dry density for the Standard Proctor test was 18.5 per cent.

Some cylinders were compacted at various moisture contents by the standard Proctor method and three consolidation specimens cut from each cylinder. For zero volume expansion, loads were added to the specimens to prevent swelling when water came in contact with the soil. For the other samples, the soil was permitted to expand a predetermined percentage after the water came in contact with the soil and then loaded to prevent further expansion. When the specimens showed no further tendency to swell (about 6 to 8 hours for a sample 3/4 inches thick) the load required to restrain the soil was determined.

A study of Fig. No. 15 shows that a small percentage of expansion greatly reduced the swelling pressures. Also an increase in the compacting moisture reduces the swelling. For this soil, namely, with Standard Proctor compaction and moisture contents greater than 23 percent, there is a small but constant swelling pressure regardless of the amount of expansion permitted. The chart also indicates that, to reduce the effect of swelling clays, the soils should be placed well on the "wet side" of the density curve and permitted to expand slightly when taking up moisture. It may not be possible to permit expansion of the soil for some structures, but if that is true then one must expect the higher expansion pressures. The writer firmly believes that flexible structures which, like a boxer, "roll with the punches," will be subject to much less damage due to swelling soils than will the more rigid structures.

A study of the swelling pressures of this soil when compacted at densities lower than Standard Proctor has not been successful because of the non-uniformity of the specimens.

The authors suggest pre-wetting as a method of controlling expansion. This offers possibilities; but the question is: how long must the soil be pre-wetted? In the housing project in Austin the soil under one of the houses was pre-wetted before construction. Trenches dug for the foundations were filled with water which was allowed to stand approximately six weeks. Then the water was pumped out, the foundations placed on the wet soil, the trenches again filled with water and the soil kept saturated thereafter. The house, like those constructed in the dry trenches, has continued to rise since construction. Some of these houses have been built for over six years and are still rising. The question of how long the soil must be pre-wetted before this method will be successful has not yet been answered.

The writers give some very interesting field experiences that will be valuable to engineers designing and building in materials of this type. They report a maximum upward movement of slightly greater than one foot. In San Antonio, a basement floor was placed on a slab on fill, and the slab had actually raised 1.5 ft. above the original grade.

The drilled and belled, cast-in-place piling that was used on the "Wellton-Mohawk Pumping Plants" has been used successfully in the Texas area for over twenty years. A large truck-mounted earth auger drills a vertical shaft to a depth which is below the line of seasonal moisture change and a reaming attachment bells out the bottom of the pile. This bell is very essential to this type of construction because the expanding soil will often grip the pile and

lift it. If a straight shaft pile is used, it can easily be lifted: however, the bell offers great resistance to this vertical lifting. The writer knows of no underreamed piles, properly constructed and reinforced, that have been damaged by the vertical pressures due to the lifting of the piles. Nevertheless, there has been damage to this type of foundation when it has been placed on the hillside. Here hillside creep has broken the piles and damaged structures. When construction on this type of soil is placed on hillsides extreme care must be taken in order to obtain a satisfactory foundation.

While only a limited number of thorough examinations have been made, it has been found that piers and columns are usually sheared off at a depth of four to six feet below the surface of the ground where hillside creep has occurred. Perhaps in other areas this depth may be different from those the writer has observed.

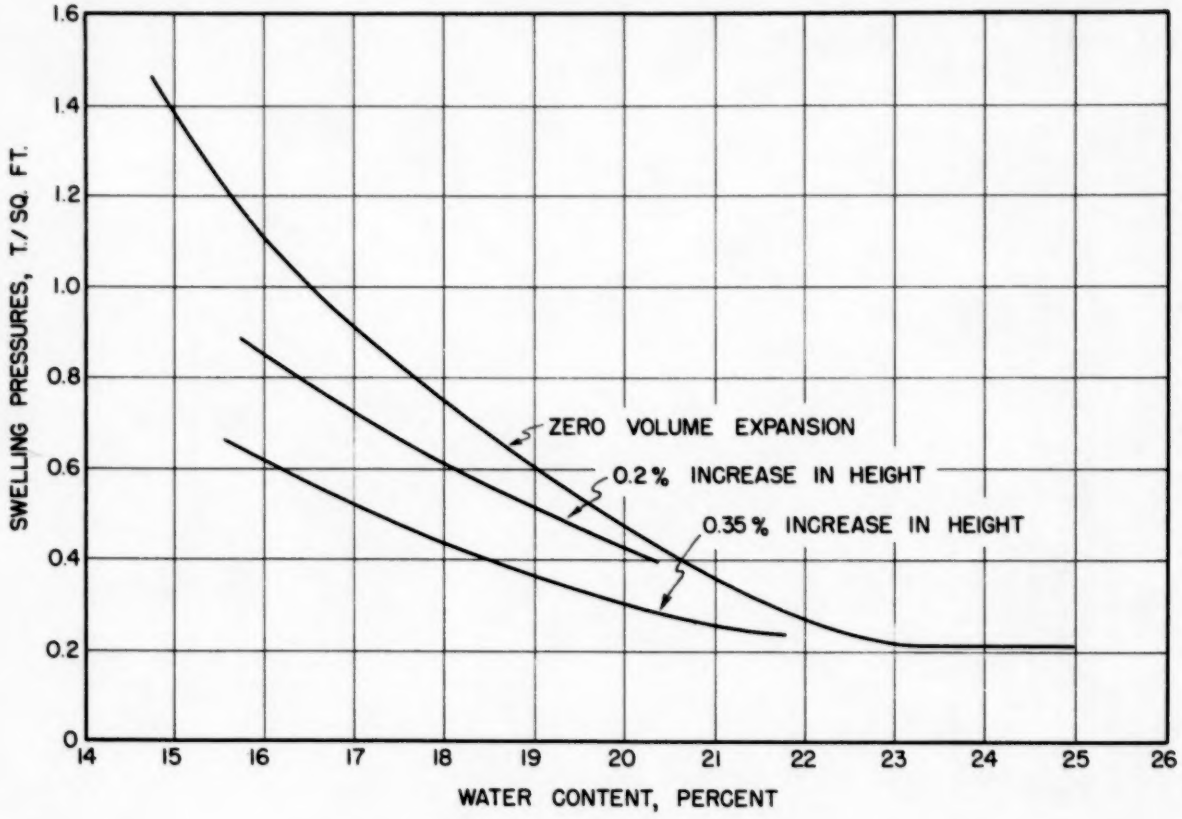

FIG. NO. 15. RELATION OF SWELLING PRESSURE TO MOISTURE CONTENT

Discussion of
"FOUNDATION TREATMENT FOR EARTH DAMS ON ROCK"

by Thomas F. Thompson
(Proc. Sep. 548)

THOMAS W. FLUHR,[1] M. ASCE.—The paper by Mr. Thompson serves a useful purpose in calling attention to the problem of obtaining an adequate foundations for earth dams. As stated, no two sites offer the same problem. Of particular importance is the statement that rock surfaces should be shaped so as to minimize differential settlement in the fill. Well-compacted earth fill, under such stresses, shears rather than flows, and open shear planes may be formed in the embankment. Some dam designers not only omit consideration of such differential settlement, but in fact, create the problem by introducing concrete of steel cutoff walls into the embankment. Yet most engineers will recognize the danger of placing part of a building foundation on soil and part on rock.

Most engineers have little or no conception of the effects of explosives on rock, and make little or no provision in their specifications for control of drilling and blasting. A contractor, of course, is interested in removing rock in the quickest and most economical manner, usually without consideration of the ultimate effect on the structure.

Pressure grouting of the foundations of a dam or other structure is looked on by many designers as a universal panacea. It must be recognized that relatively few engineers have had practical experience with grouting other than as a routine procedure. Anyone who has taken great trouble to do a thorough job of grouting a rock foundation, and later has seen the grouted rock excavated, must conclude that pressure grouting, no matter how well-done, is only a weak crutch which can help, but never totally cure, the weaknesses of a poor foundation.

Since no two sites present identical foundation problems, and since the problems are seldom appreciated until the rock has been exposed, each requires individual attention. Solution of the problems demands vast quantities of common sense. It also requires flexibility in the specifications in order to meet any eventualities. Even when the designer has adopted theoretically adequate methods of treating dam foundations, the problem still exists of obtaining thorough inspection to insure that the plans are carried out.

It is to be hoped that Mr. Thompson's paper will direct the attention of dam designers to the fact that foundation problems do exist, and that adequate treatment of them is as yet seldom attained in practice.

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Discussion of
"EXPLORATION PRINCIPLES FOR
MAJOR ENGINEERING WORKS"

by W.R. Judd
(Proc. Sep. 550)

THOMAS W. FLUHR,[1] M. ASCE.—The work of the Bureau of Reclamation has covered a greater variety of geologic conditions than that of most other engineering organizations and for that reason Mr. Judd's paper embraces more kinds of problems than most engineers in this field encounter.

Mr. Judd mentions slaking of rock by air. The writer has often seen rock in tunnel excavations deteriorate, but as yet has found no good and sufficient reason for attributing the effect to air. In the cases within the writer's experience, the deterioration appeared to have been caused by the relief of stress resulting from removal of adjacent or superincumbent material.

Mr. Judd appears to think that there may be some relation between the number and type of tunnel supports and rock types, a problem now being studied by a task force of the Committee on Engineering Geology of the A.S.C.E. The problem is not a new one, but previous attempts to solve it have not been productive. In the writer's experience there is no correlation between rock types and the use of support. He has seen tunnels driven for miles with little or no support, whereas adjacent tunnels, driven by a different contractor through the identical rock formation, of the same physical characteristics, were one hundred percent supported. The requirement for tunnel support, in most cases within the writer's experience, has been determined by the economics of the situation. If the contract makes it unprofitable to use support, the contractor conducts his operations so as to minimize support requirements. On the other hand, if he finds the use of support profitable, he drills widely-spaced holes and loads them heavily. By drilling and shooting wild a contractor can make any tunnel roof weak. Moreover, under modern tunnel-driving methods enough powder is used to break the muck small so that it can be loaded easily by mucking machines. In addition, state and governmental authorities and insurance companies have emphasized the need for safety. Hence there is greater pressure for the use of tunnel support than existed even a few years ago. The use of tunnel support therefore depends greatly on factors extraneous to the type and physical characteristics of the rock.

The writer disagrees with the statement by the author that "It rather has attempted to establish principles by which the average geologist untrained in engineering geology can intelligently and safely develop exploration programs for such structures." Even an above-average geologist, untrained in engineering geology, and even if trained, lacking in experience, is of little value in such work.

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